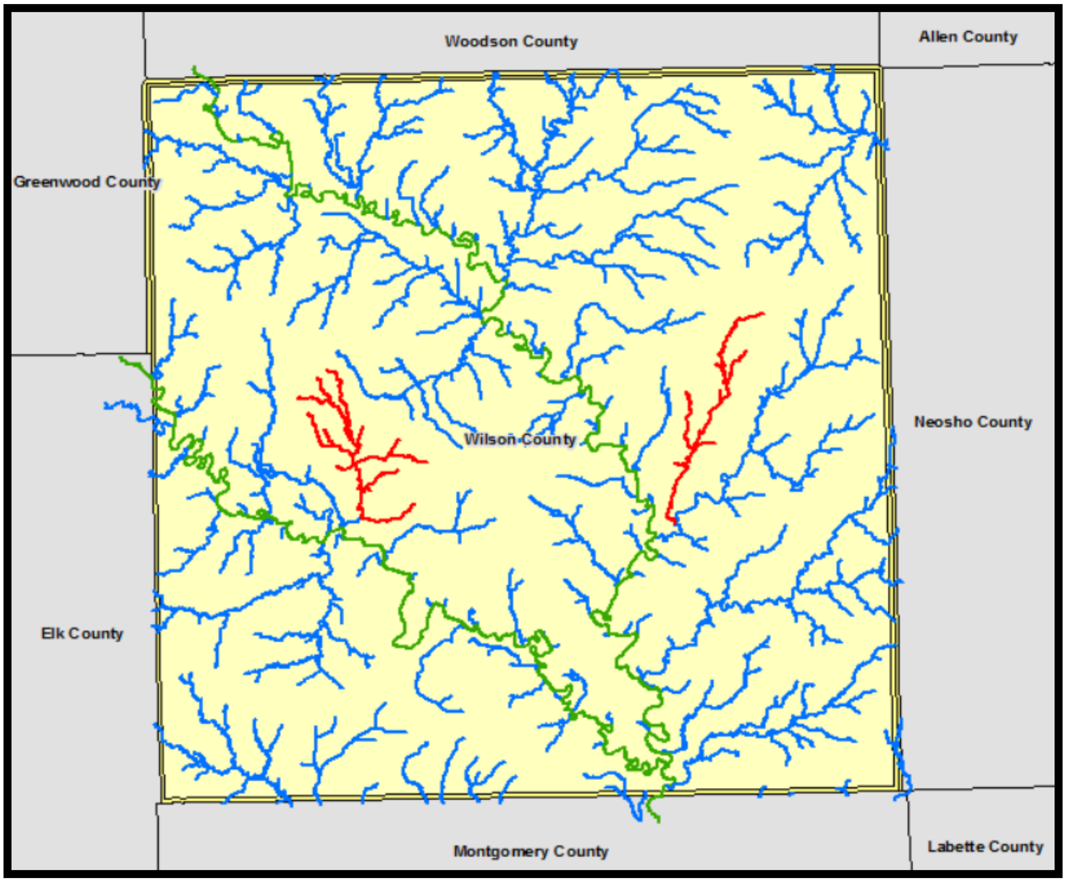
HYDROLOGY REPORT
WILSON COUNTY, KANSAS



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PREPARED BY:
AMEC FOSTER WHEELER ENVIRONMENT & INFRASTRUCTURE INC.
100 SE 9TH STREET, SUITE 400
TOPEKA, KS 66612

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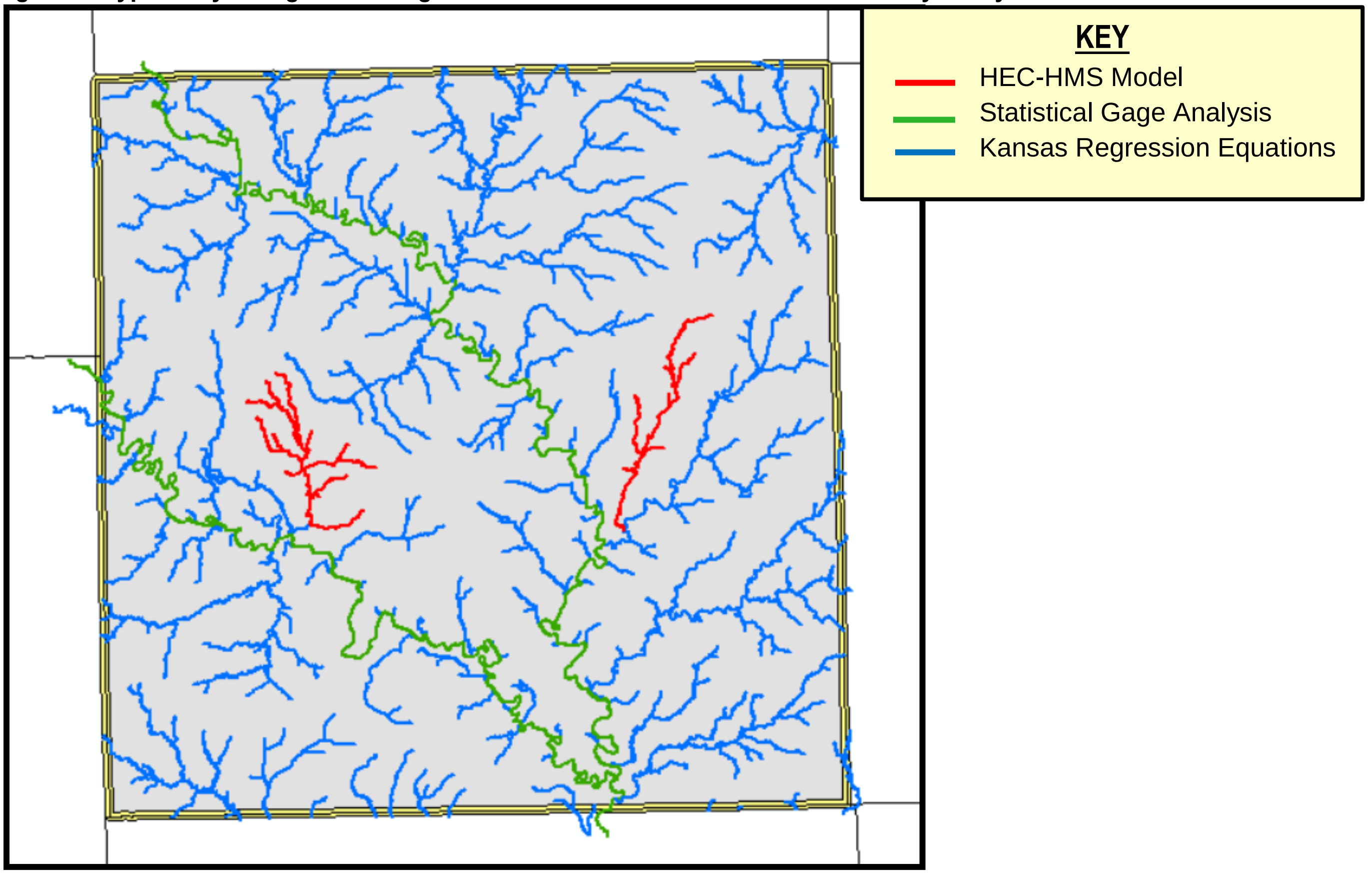
INTRODUCTION

This report presents the hydrologic analyses for the detailed Zone AE streams, the enhanced Zone AE designated streams, and the approximate Zone A designated streams in Wilson County, Kansas. This project consists of new hydrologic and hydraulic studies using current watershed characteristics and new detailed topography. This study includes approximately 8.3 miles of streams modeled by detailed methods, resulting in updated Zone AE floodplains with a floodway; approximately 9.9 miles of streams modeled by enhanced methods, including rainfall-runoff model hydrology and field measured structures, resulting in updated Zone AE floodplains without a floodway; and approximately 870.4 miles of streams studied by approximate methods, resulting in updated Zone A floodplains. Enhanced hydrology was performed on approximately 44.4 miles of streams, including a number of approximate study streams within the Little Cedar Creek and Salt Creek watersheds, using rainfall-runoff models. In addition, statistical gage analysis was performed for approximately 115.8 miles of streams, including the detailed Zone AE stream segments and the Zone A stream segments of the Fall River and Verdigris River. For streams not included in an enhanced hydrology model or analysis, Zone A stream hydrology was performed using USGS Rural Regression Equations for Kansas. A summary of the streams that were studied is shown in Table 1. A figure that shows the type of hydrologic method used for each stream is shown in Figure 1.

Table 1: Summary of Methods		
Study Area/Flooding Source	**Stream Miles**	**Hydrologic Method**
Fall River	51.6	Statistical Gage Analysis
Little Cedar Creek	2.7	Rainfall-Runoff Model (HEC-HMS)
Salt Creek	2.6	Rainfall-Runoff Model (HEC-HMS)
Salt Creek Tributary 1	2.4	Rainfall-Runoff Model (HEC-HMS)
Salt Creek Tributary 3	1.7	Rainfall-Runoff Model (HEC-HMS)
Verdigris River	64.2	Statistical Gage Analysis
Various Zone A Streams within Little Cedar Creek and Salt Creek Watersheds	35.0	Rainfall-Runoff Model (HEC-HMS)
Various Zone A Streams	728.4	Kansas Regression Equations
Total	**888.6**	-

Figure 1- Type of Hydrologic Modeling Used for Each Stream in the Wilson County Study



This hydrologic study was performed to develop peak discharges for the 10%, 4%, 2%, 1%, 1%-minus, 1%-plus and 0.2% annual chance storm events. The peak discharges computed from this analyses will be used in developing the hydraulic analyses for the streams within this study.

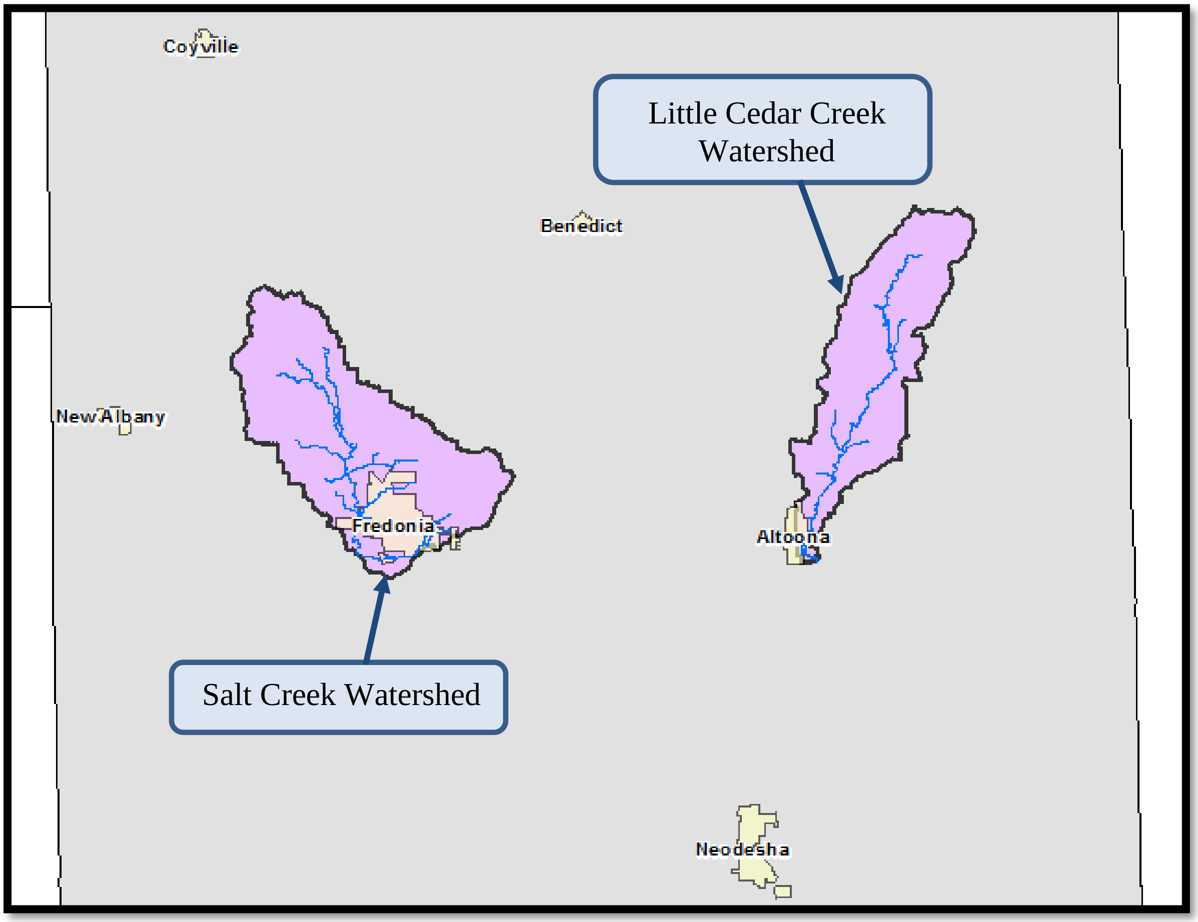
The extents of the Zone A studies include those streams currently designated by FEMA, up to a drainage are of 0.4 square miles, plus the conveyances with drainage areas equal to or greater than 1-square mile of drainage area. A detailed adjustment of the stream network relative to aerial photography and LiDAR was completed to ensure proper streamline alignment and extent.

There is no current county-wide FIS Report for Wilson County. There is a current FIS Report for the City of Neodesha, which is dated February 1978.

General Rainfall-Runoff Model

The rainfall-runoff model HEC-HMS version 4.2 (Reference 2), developed by the USACE, was used for the two detailed rainfall-runoff models within this project, which include the Little Cedar Creek watershed and the Salt Creek watershed. Figure 2 shows the extent of these two rainfall-runoff models. Amec Foster Wheeler used HEC-HMS to generate subbasin runoff hydrographs for the 10%, 4%, 2%, 1%, 1%-minus, 1%-plus and 0.2% chance 24-hour SCS Type II rainfall events. These runoff hydrographs were routed and combined along the studied streams to produce the peak discharges.

Figure 2: Boundaries of the Little Cedar Creek Watershed and Salt Creek Watershed HEC-HMS Models



Subbasin boundary delineations were based on topography obtained as 1-meter LiDAR through the Kansas Data Access and Support Center (DASC). Subbasin boundaries were first delineated using automated GIS processes including HEC-GeoHMS (Reference 3) and ArcHydro (Reference 4) based on LiDAR Digital Elevation Models (DEM), and then manually edited as needed based on storage considerations and the most recent aerial photography available.

The towns partially encompassed within the HEC-HMS models, Fredonia and Altoona, have minimal storm water drainage systems. Furthermore, the majority of the storm water drainage systems they do have were only designed to contain runoff from the smaller storm events, generally the 10% annual chance event or smaller. The primary purpose of this mapping update is to accurately model the risk associated with the larger storm events, specifically the 1% annual chance and 0.2% annual chance flooding events. During these larger storm events, surface water does not necessarily follow the sub-surface flows of the storm water drainage systems. Therefore, the storm water drainage networks (storm sewers) were not included in the HEC-HMS models as they are considered insignificant for the larger storm events and for this particular study.

RAINFALL

The rainfall depths, shown in Table 2, were computed using rainfall grids developed by NOAA as part of Atlas 14: Precipitation-Frequency Atlas of the United States (Reference 5). The depths represent an average of all partial-duration grid values within the areas that are included in the rainfall-runoff models. The 1%-minus and 1%-plus rainfall depths were computed by using the 1% annual-chance rainfall depth, the 95% lower confidence limit depth, and the 95% upper confidence limit depth published in Atlas 14; along with the known sample size of 1,000 data sets used in Atlas 14; to compute the standard deviation. This computed standard deviation was then used to calculate the 16% lower and 84% upper confidence limits, which are the values used for the 1%- minus and 1%-plus rainfall depths, respectively.

Table 2: SCS Type II 24-hour Rainfall Depths for HEC-HMS Models

Event (annual-chance)	Little Cedar Creek Watershed Depth (inches)	Salt Creek Watershed Depth (inches)
10%	5.6	5.5
4%	7.0	6.9
2%	8.2	8.1
1%	9.4	9.3
1%-minus	8.3	8.2
1%-plus	10.7	10.7
0.2%	12.8	12.8

Rainfall values were also computed using the annual-maximum series. A comparison of these rainfall values to the partial-duration series is shown in Tables 3 and 4. Since the calculations for the annual-maximum series rely on only one flood event for each year, and since the lower storm events are more likely to have multiple flood events in a given year, the partial-duration series would be more appropriate for lower frequency events. In addition, since the two values are predominately the same for the higher storm events, it was determined that the partial-duration rainfall values would be appropriate for all storm events in this study.

Table 3: Comparison of Rainfall for Partial-Duration and Annual-Maximum Series for Little Cedar Creek Watershed

Event (annual-chance)	Partial-Duration Series			Annual-Maximum Series		
	Minimum (in)	Mean (in)	Maximum (in)	Minimum (in)	Mean (in)	Maximum (in)
10%	5.6	5.6	5.7	5.5	5.6	5.6
4%	7.0	7.0	7.0	6.9	7.0	7.0
2%	8.1	8.2	8.2	8.1	8.1	8.2
1%	9.4	9.4	9.4	9.4	9.4	9.4
1% lower	7.2	7.2	7.2	7.2	7.2	7.2
1% upper	12.0	12.0	12.0	11.9	12.0	12.0
0.2%	12.8	12.8	12.8	12.8	12.8	12.8

Table 4: Comparison of Rainfall for Partial-Duration and Annual-Maximum Series for Salt Creek Watershed						
Event	Partial-Duration Series			Annual-Maximum Series		
	Minimum (in)	Mean (in)	Maximum (in)	Minimum (in)	Mean (in)	Maximum (in)
10%	5.5	5.5	5.5	5.4	5.5	5.5
4%	6.9	6.9	6.9	6.8	6.9	6.9
2%	8.0	8.1	8.1	8.0	8.0	8.1
1%	9.3	9.3	9.4	9.3	9.3	9.4
1% lower	7.1	7.1	7.2	7.1	7.1	7.2
1% upper	11.9	12.0	12.0	11.9	11.9	12.0
0.2%	12.7	12.8	12.8	12.7	12.8	12.8

Rainfall Loss

The U.S. Department of Agriculture Soil Conservation Service (SCS) Curve Number Method was used to model rainfall loss (Reference 8). The curve number (CN) is a function of both hydrologic soil group and land use. The table used to determine the CN value from the soil hydrologic soil group and land use is included as Table 5. The CN tables used assume an antecedent runoff condition (ARC) of II as it is representative of typical conditions, rather than the extremes of dry conditions (ARC I) or saturated conditions (ARC III).

The value for initial abstraction was left blank in the HMS input file. Per the HMS documentation, doing so will cause the program to calculate the initial abstraction as 0.2 times the maximum potential retention (S) which is calculated from the CN as $S = (1000/CN) - 10$. This method is based on empirical relationships developed from the study of many small experimental watersheds, and is a commonly accepted method of estimating the initial abstraction.

Soils Data

Soils data was obtained in shapefile and database format from the United Stated Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) website (Reference 6). Typical soils in the study area consist primarily of hydrologic soil group D.

Land Use

Land use was determined using a combination of data from the National Land Cover Dataset (NLCD) website (Reference 7) and aerial photography. Fifteen land use designations were utilized to develop the CN values for each subbasin. The CN values were taken from “TR-55 Urban Hydrology for Small Watersheds” Table 2-2 (Reference 8). The land use designations are shown in Table 5. As previously mentioned, the CN values were calculated using ARC II conditions, as represented in Table 5.

Table 5: CN Land Use and Soil Drainage Class Table				
	Weighted CN (Includes Impervious)			
Land Use Description	A	B	C	D
Open Water	98	98	98	98
Developed, Open Space	51	68	79	84
Developed, Low Intensity	57	72	81	86
Developed, Medium Intensity	77	85	90	92
Developed, High Intensity	89	92	94	95
Barren Land	77	86	91	94
Deciduous Forest	30	55	70	77
Evergreen Forest	30	55	70	77
Mixed Forest	30	55	70	77
Shrub/Scrub	43	65	76	82
Herbaceous	43	65	76	82
Hay/Pasture	49	69	79	84
Cultivated Crops	65	75	82	86
Woody Wetlands	36	60	73	79
Emergent Herbaceous Wetlands	36	60	73	79

The soil and land use data were combined using GIS processes in which specific CNs were defined for each soil-land use relationship shown in Table 5. Area-weighted CN values were computed for each subbasin using GIS processes. The area-weighted CN values were used in the HEC-HMS models.

RAINFALL TRANSFORM (HYDROGRAPH)

The time of concentration for each subbasin was calculated using the methodology outlined in TR-55 Urban Hydrology for Small Watersheds (Reference 8) and Chapter 15: Time of Concentration of the National Engineering Handbook (Reference 9). A GIS process was utilized to calculate the longest flow path within any given subbasin. The longest flow paths were then manually edited based on topographic data and visual inspection of aerial photography to produce an effective time of concentration line. The total time of concentration consists of the sum of the travel times for sheet flow, shallow concentrated flow, and channel flow. Based on information described in TR-55 Urban Hydrology for Small Watersheds (Reference 8), the maximum sheet flow length is approximately 300 feet. The areas within the HEC-HMS models are rural areas with moderate slopes. Therefore, it was determined that a maximum sheet length of 200 feet was appropriate for the majority of the subbasins in the model, with the exception of a few subbasins that warranted slightly longer or slightly shorter sheet lengths. The division between shallow concentrated flow and channel flow was defined based on watershed features exhibited on the aerial images and topography. In certain situations, it was necessary to define multiple shallow concentrated and channel flow regimes for a given longest flow path. Time of concentration over water bodies was calculated using wave velocity.

The parameters of flow area and wetted perimeter are required inputs for calculating the flow velocity used in the channel time of concentration calculations. Typical channel cross sections were defined for each subbasin, and trapezoidal cross-sections were defined from the project topography. In order to calculate the flow area and wetted perimeter, several factors need to be considered. For open channel flow, a trapezoidal channel shape was selected based on examination of aerial photography and topography. Channel width was approximated by close visual inspection of the aerial photography and LiDAR topography.

The runoff was transformed into a hydrograph using the SCS Unit Hydrograph method. This method makes use of lag time, which is estimated as 0.6 times the time of concentration. Moderate relief is present in the study area; therefore, surface storage attenuation does not generally need to be accounted for in typical subbasins. Thus, it was determined that the SCS Unit Hydrograph is the most appropriate transform method to use for this study area.

ROUTING

The Muskingum-Cunge channel routing method was used for routing runoff through all reaches in the model. The channel geometry, slope, and hydraulic roughness were assigned, based on the LiDAR data and the aerial images. Eight-point cross sections were developed, based on examination of aerial photography and topography. Manning's channel roughness values for the routing reaches were selected based off the aerial photography.

LITTLE CEDAR CREEK WATERSHED

The HEC-HMS model of the Little Cedar Creek watershed has a total drainage area of approximately 13.1 square miles. The model includes 27 subbasins, ranging from 30 to 1,061 acres. Three of the subbasins contain residential areas within the city of Altoona, while the remaining areas are predominately rural.

Rainfall and Aerial Reduction

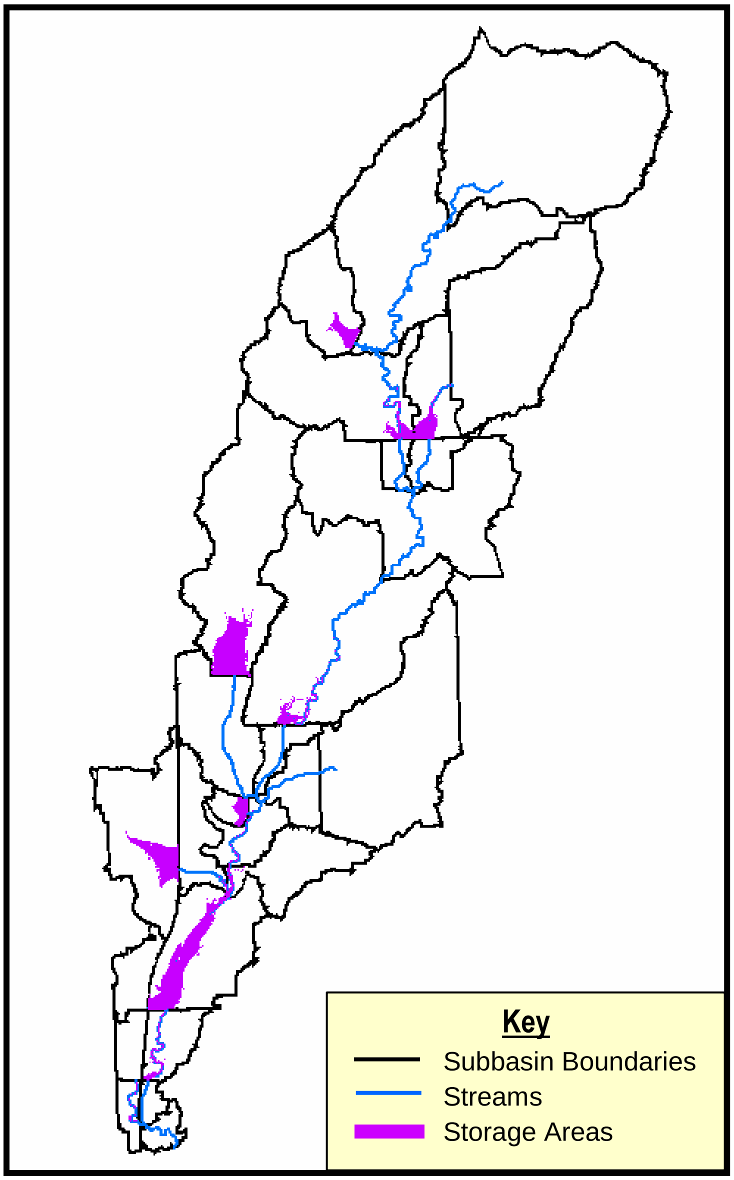
Areal reduction of the point rainfall depths was not deemed necessary for the Little Cedar Creek watershed study since the rainfall depths were generated using the watershed boundary.

Storage Routing

Ten storage areas were modeled in the Little Cedar Creek watershed hydrologic model. Three of the storage areas represent storage behind significant dams located within the watershed, and seven storage areas represent storage behind significant road embankments within the watershed. The criteria for including storage areas within the model was based on the storage type and the storage volume. Specifications for dam tops, associated spillways, and associated outlet structures were included in the HEC-HMS model, where applicable. As-built plan information obtained from the Kansas Department of Agriculture was used for the outlet structures, spillways, and dam tops for the two state permitted dams. Information on the outlet structures and dam tops of the storage areas behind road embankments were obtained using LiDAR topography and aerial imagery, as was the information for the non-permitted dam. Depth-storage rating curves were estimated from LiDAR topography, assuming LiDAR elevations represent normal pool, using an automated area-volume tool within GIS, at a minimum of 0.5-foot intervals.

Figure 3, illustrates the extent of the maximum water elevation during the 1% annual-chance storm event for all the storage areas included in the Little Cedar Creek watershed HEC-HMS model, along with subbasin boundaries and streamlines.

Figure 3- Extent of Maximum Water Elevation of Modeled Storage Areas during 1% chance storm event for the Little Cedar Creek Watershed.



SALT CREEK WATERSHED

The HEC-HMS model of the Salt Creek watershed has a total drainage area of approximately 19.0 square miles. The model includes forty-five subbasins, ranging from 39 to 897 acres. Fifteen of the subbasins contain residential and urbanized areas within the city of Fredonia, while the remaining areas are predominately rural.

Rainfall and Areal Reduction

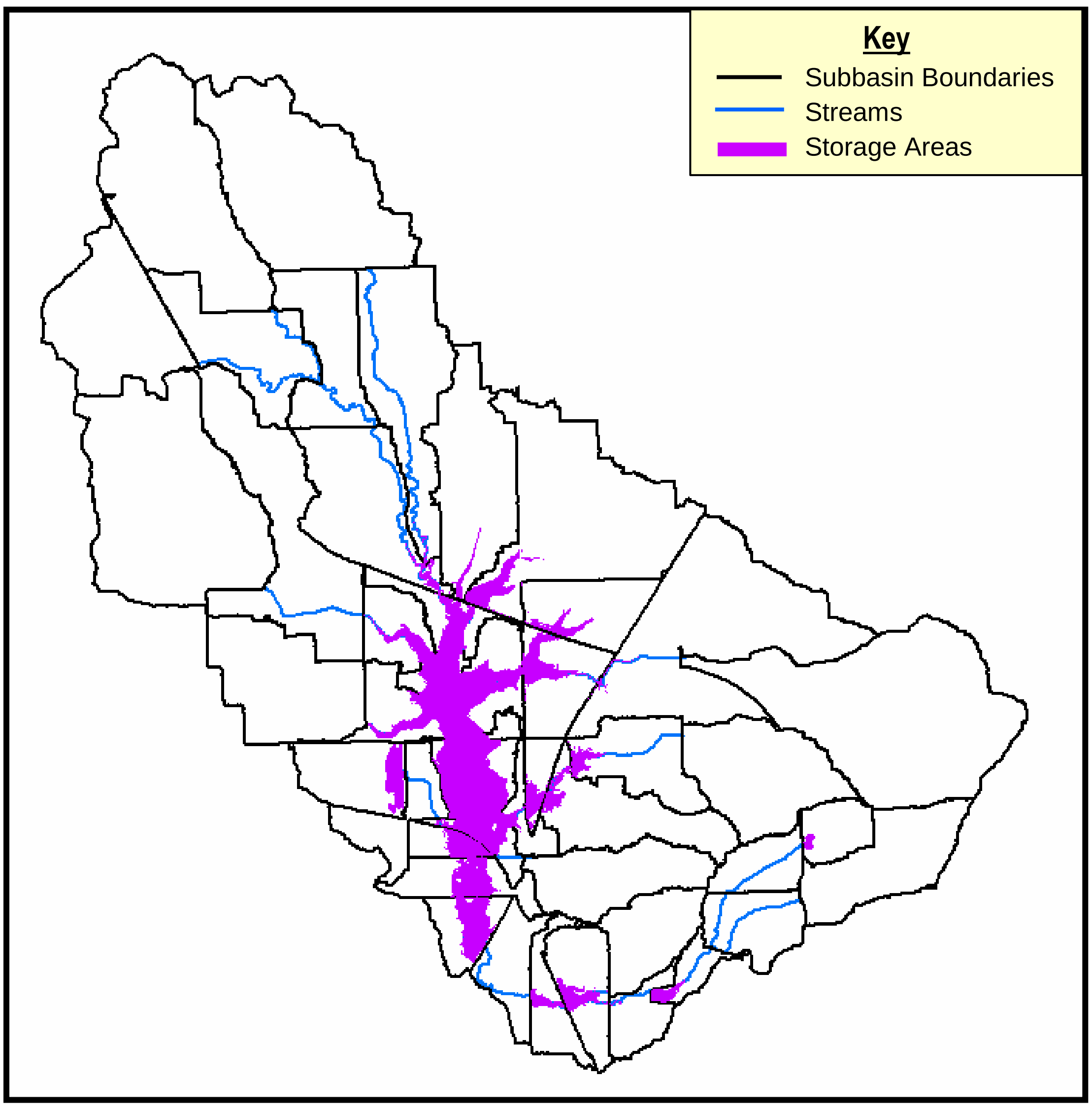
Areal reduction of the point rainfall depths was not deemed necessary for the Salt Creek watershed study since the rainfall depths were generated using the watershed boundary.

Storage Routing

Eighteen storage areas were modeled in the Salt Creek watershed hydrologic model. Two of the storage areas represent storage behind significant dams located within the watershed, and sixteen storage areas represent storage behind significant road/railroad embankments within the watershed. The criteria for including storage areas within the model was based on the storage type and the storage volume. Specifications for dam tops, associated spillways, and associated outlet structures were included in the HEC-HMS model, where applicable. Information on the dam tops, spillways, and outlet structures were obtained using LiDAR topography and aerial imagery. Depth-storage rating curves were estimated from LiDAR topography using an automated area-volume tool within GIS, at a minimum of 0.5-foot intervals.

Figure 4, illustrates the extent of the maximum water elevation during the 1% annual chance storm event for all the storage areas included in the Salt Creek watershed HEC-HMS model, along with subbasin boundaries.

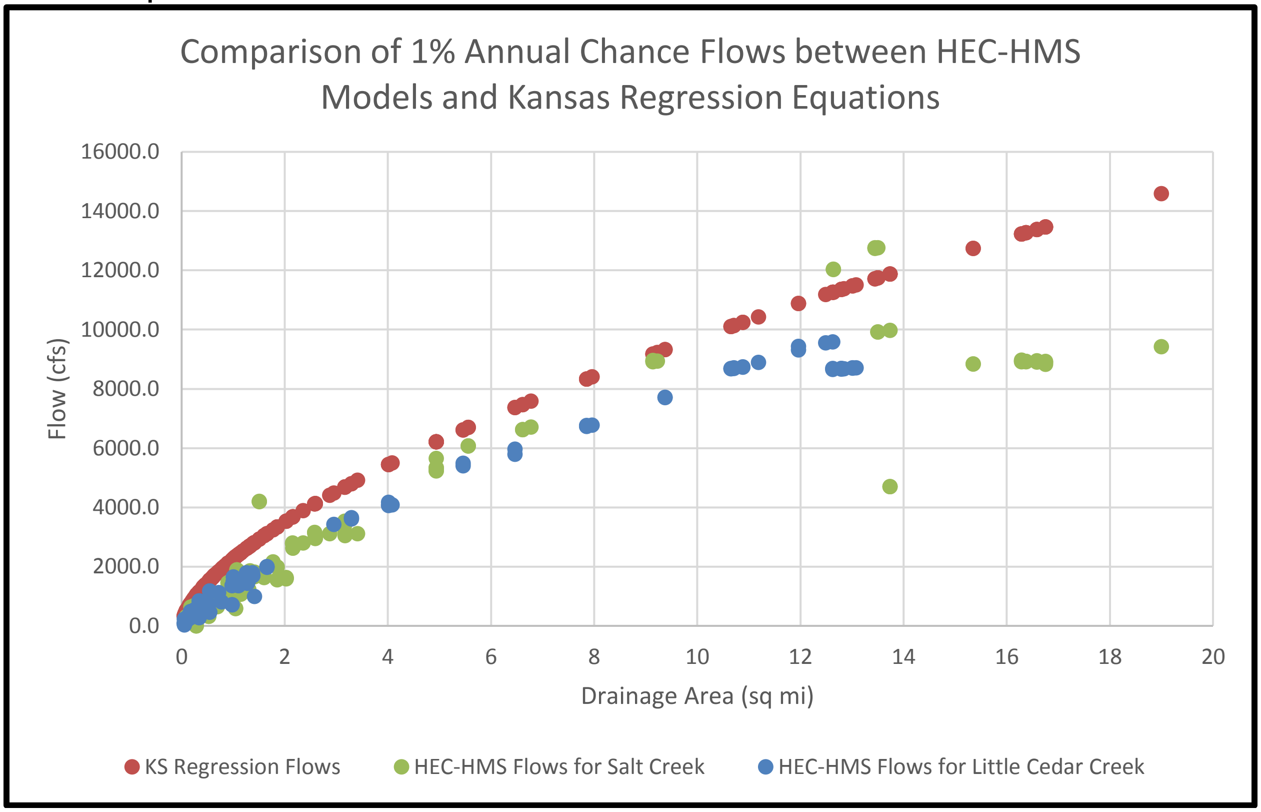
Figure 4- Extent of Maximum Water Elevation of Modeled Storage Areas during 1% chance storm event for the Salt Creek Watershed.



FLOW COMPARISON

There is not an effective FIS Report for the City of Altoona, Kansas or the City of Fredonia, Kansas. The peak discharges from the HEC-HMS models were compared to the peak discharges calculated using the Kansas Regression Equations. Figure 5 shows a comparison between the 1% annual chance flows from the HEC-HMS models and the 1% annual chance Kansas Regression Flows. The majority of the flows from the HEC-HMS models fall slightly under the Kansas Regression Flows, but are within an acceptable tolerance.

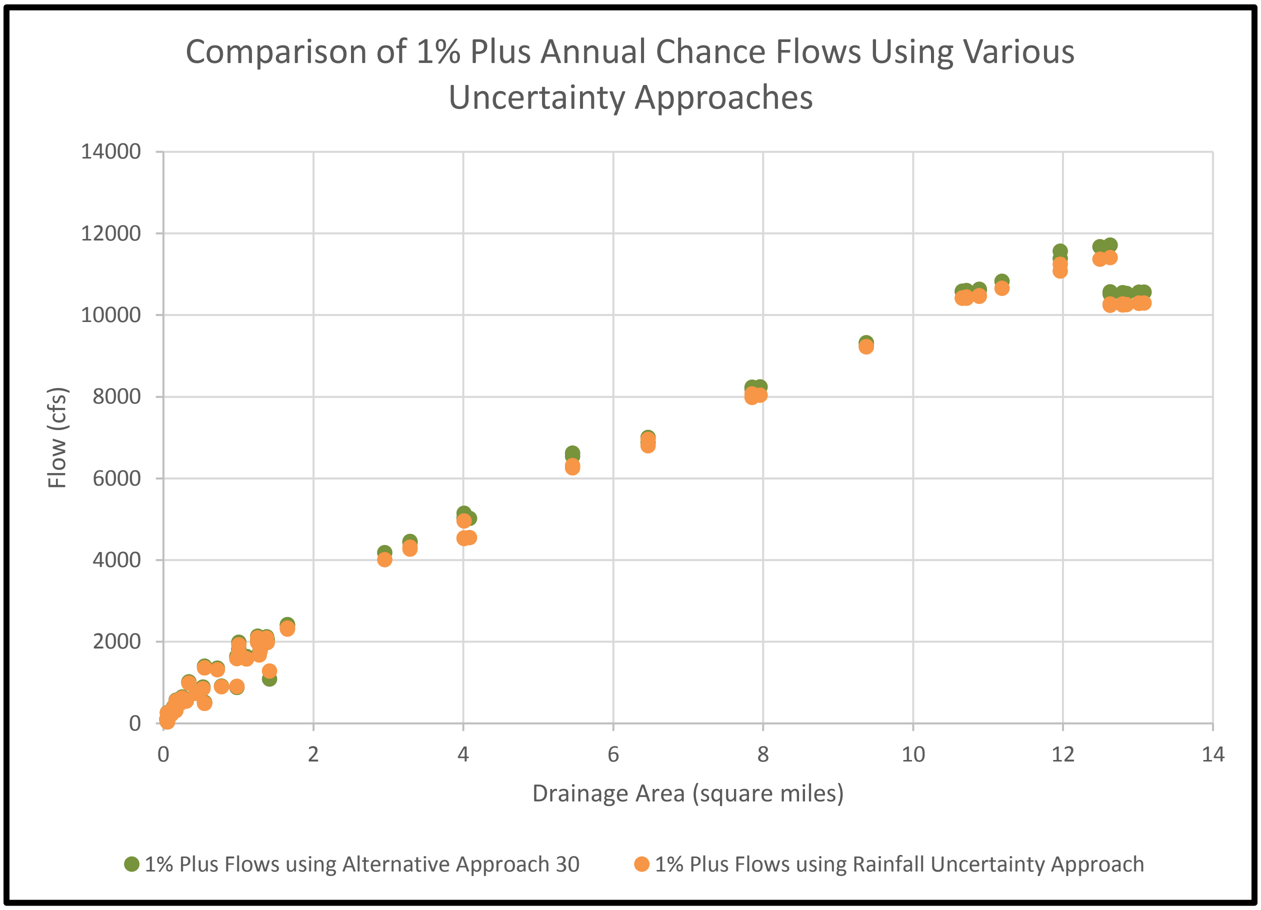
Figure 5- Comparison of 1% Annual Chance Flows between HEC-HMS Models and Kansas Regression Equations.



The 1% plus annual chance flows generated by the HEC-HMS models, which accounts for variability that exists in the statistics of the rainfall calculations by using a 1% plus rainfall depth, were compared to the 1% plus annual chance flows calculated using an alternative method that combines the procedures described in Bulletin 17B (Reference 10) and the US Army Corps of Engineer's Risk-Based Analysis for Flood Damage Reduction Studies Engineer Manual (Reference 17), which utilizes the 50%, 10%, and 1% annual chance peak flows from the HEC-HMS models and an equivalent record length. Figure 6 shows the comparison between the two different uncertainty approaches. The calculations for the alternative uncertainty approach uses an equivalent record length of 30 years, which is an appropriate equivalent record length for calibrated rainfall-runoff models based on the guidance. The 1% plus annual chance flows using the rainfall uncertainty approach are nearly identical to the 1% plus annual chance flows using the alternative uncertainty approach with an equivalent record length of 30 years. While 30 is documented as the maximum equivalent record length to be used in the calculations, it still falls within an appropriate range for the modeling done and aligns with the 1% plus annual chance flows generated by the

HEC-HMS model, using the 1% plus rainfall depth. Therefore, it was deemed appropriate to utilize the rainfall uncertainty approach to determine the 1% plus annual chance flows for the streams included in the HEC-HMS models for this project.

Figure 6- Comparison of 1% Plus Annual Chance Flows Using Various Uncertainty Approaches

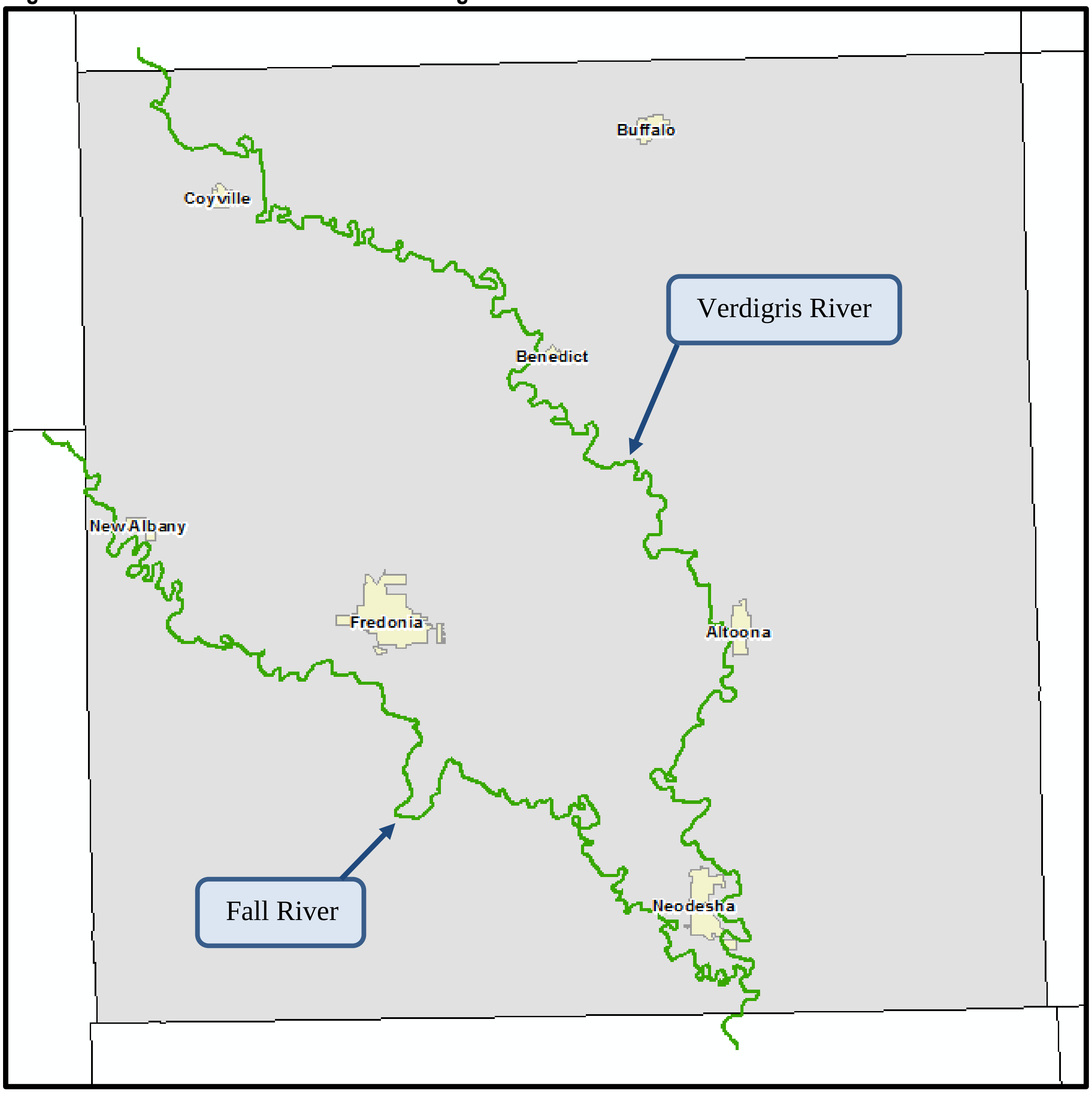


GAGE ANALYSIS

Five USGS gage stations were analyzed as part of this study. Three of the gages analyzed are on the Verdigris River; one near Coyville, Kansas; one near Altoona, Kansas; and one at Independence, Kansas. The gage near Coyville is located at Decatur Road. The gage near Altoona is located at East Washington Street. The gage at Independence, which is downstream of the area included in this study, is located at East Main Street. Two of the gages analyzed are on the Fall River; one near Fall River, Kansas and one at Fredonia, Kansas. The gage near Fall River, which is upstream of the area included in this study, is located at 27th Street, just below the Fall River Lake dam. The gage at Fredonia is located at Harper Road, just upstream of the confluence with Clear Creek. A summary of these five gages is shown in Table 6. Annual peak flow records were obtained from the USGS Water Resources website (Reference 15). All five gages have significant period of record in which a confident peak flow frequency analysis could be computed. Figure 7 shows the locations of the Fall River and Verdigris River within Wilson County.

Table 6: Summary of USGS Stream Gages			
USGS Gage Number	Gage Description	Drainage Area (mi^2)	Period of Record
07168500	Fall River near Fall River, KS	585	1939-1989
07169500	Fall River at Fredonia, KS	827	1923-2015
07166000	Verdigris River near Coyville, KS	747	1940-1998
07166500	Verdigris River near Altoona, KS	1,094	1939-2015
07170500	Verdigris River at Independence, KS	2,892	1904-2015

Figure 7: Location of the Fall River and Verdigris River Extents



Gage analyses were performed on these USGS gages using Bulletin 17C parameters (Reference 11), utilizing the USACE HEC-SSP software (Reference 12). An upper confidence limit of 84% was used to determine the flows for the 1% plus annual chance storm event. A lower confidence limit of 16% was used to determine the flows for the 1% minus annual chance storm event.

USGS 07168500- Fall River near Fall River, KS
USGS Station 07168500 is located near Fall River, Kansas and has 52 years of record, dating from 1939 to 1989. Construction of the Fall River Lake dam was completed in 1949. This gage is located just downstream of the Fall River Lake dam, at 27th Street, but is located upstream of the auxiliary spillway inlet channel. Therefore, any flow exiting Fall River Lake through the auxiliary spillway is not included in the peak discharge at the gage. Thus, frequency flow estimates calculated for this site were not utilized in determining peak discharges for the Fall River.

USGS 07169500- Fall River at Fredonia, KS
USGS Station 07169500 is located at Fredonia, Kansas and has 81 years of record, dating from 1923 to 2015. Construction of the Fall River Lake dam, which is located upstream of the study area for this project, was completed in 1949. Therefore, frequency flow estimates were calculated for this site, using only the time period after 1949.

USGS 07166000- Verdigris River near Coyville, KS
USGS Station 07166000 is located near Coyville, Kansas and has 59 years of record, dating from 1940 to 1998. Construction of the Toronto Lake dam, which is located just upstream of the study area for this project, was completed in 1960. Therefore, frequency flow estimates were calculated for this site, using only the time period after 1960.

USGS 07166500- Verdigris River near Altoona, KS
USGS Station 07166500 is located near Altoona, Kansas and has 77 years of record, dating from 1939 to 2015. Construction of the Toronto Lake dam was completed in 1960. Therefore, frequency flow estimates were calculated for this site, using only the time period after 1960.

USGS 07170500- Verdigris River at Independence, KS
USGS Station 07170500 is located at Independence, Kansas and has 100 years of record, dating from 1904 to 2015. Construction of the Fall River Lake dam, completed in 1949, and the Toronto Lake dam, completed in 1960, both impacted the flow of the Verdigris River at Independence. Therefore, frequency flow estimates were calculated for this site, using only the time period after 1960. It should be noted that this gage is downstream of the study area for this project.

Statistical Gage Analysis Results

A station and weighted skew was evaluated for the four gages selected for the statistical gage analysis. A regional skew was not evaluated as part of this analysis, as all four gages have significant period of record for a confident peak flow frequency analysis. Table 7 shows a comparison of the 1% annual chance storm event, using the two methods of skew.

Table 7: 1% Annual-Chance Comparison of Skew Methods			
USGS ID	DA (sq mi)	Station Skew (cfs)	Weighted Skew (cfs)
07169500	827	51,500	45,894
07166000	747	13,300	10,725
07166500	1,094	56,856	50,178
07170500	2,892	130,919	109,980

When considering the length of the period of record for each gage and the fact that both rivers are controlled by upstream flood control dams, it was determined that the station skew method is the most appropriate skew to use for determining peak discharges at the Fredonia, Coyville, and Altoona gages. The Fall River and Verdigris River converge upstream of the Independence gage, as does the Elk River and Verdigris River. When comparing the results from the two skew methods to previous FEMA studies conducted for Montgomery County in recent years, it was determined that the weighted skew method results more closely align to peak discharges used in those recent studies. Therefore, it was deemed appropriate to use the weighted skew results for determining peak discharges at the Independence gage.

FALL RIVER

The effective FIS Report for the City of Neodesha, Kansas lists flows for the Fall River at the Missouri Pacific Railroad, indicating a drainage area of 884 square miles. As previously mentioned, the gage at Fall River is located just downstream of the Fall River Lake dam, but is located upstream of the auxiliary spillway inlet channel. Therefore, any flow exiting Fall River Lake through the auxiliary spillway is not included in the peak discharges at the gage. Thus, the Fall River gage was not used when determining peak discharges for the Fall River. The results from the gage analysis for only the Fredonia gage were interpolated and extrapolated to produce flows at various locations along the Fall River. The station skew method results were chosen for the gage, as previously described. The Controlled Segment Interpolation Procedure was used for determining the flows along Fall Creek, as it was the most appropriate method available for the conditions of the stream. Various methods for interpolation of the flows were analyzed; including the Drainage Transfer Method, the Uncontrolled Segment Interpolation Procedure for one gage, the Controlled Segment Interpolation Procedure for one gage, and utilization of localized regression equations.

The Controlled Segment Interpolation Procedure for one gage was utilized to interpolate flows along the portion of Fall Creek that is within Wilson County. The flows were computed using the following parameters; which are described in Table 4 of the USGS Estimates of Flow Duration, Mean Flow, and Peak-Discharge Frequency Values for Kansas Stream Locations (Reference 13); utilizing flows from only the Fredonia gage.

$Q_s = (Q_u / DA_u)^* DA_s$ OR $Q_s = (Q_d / DA_d)^* DA_s$

Where:

Q_s = peak discharge at the ungaged drainage point of interest, in cubic feet per second

Q_u = peak discharge at the upstream gage location, in cubic feet per second

Q_d = peak discharge at the downstream gage location, in cubic feet per second

DA_s = total area that contributes runoff to the ungaged drainage point of interest, in square miles

DA_u = total area that contributes runoff to the upstream gage location, in square miles

DA_d = total area that contributes runoff to the downstream gage location, in square miles.

Table 8, represents the peak discharges computed as part of this statistical analysis, which incorporates analysis from the Fall River gage at Fredonia.

Table 8: Statistical Analysis Results for the Fall River

Location	Drainage Area (sq. mi.)	Peak Annual-Chance Discharges (CFS)						
		10%	4%	2%	1%	1%-	1%+	0.2%
Approximately 1.5 miles upstream of the Western Wilson County Line	672.3	18,319	26,081	33,275	41,866	33,465	62,344	68,746
At Confluence with Indian Creek	745.5	20,313	28,920	36,898	46,425	37,108	69,132	76,231
USGS Gage near Fredonia	827.0	22,534	32,082	40,932	51,500	41,165	76,690	84,565
At Confluence with Fall River Tributary 3	884.8	24,109	34,324	43,793	55,099	44,042	82,050	90,475

The peak flows listed for the Fall River in the current effective FIS Report for Neodesha, Kansas are lower than the flows described in this statistical analysis. For example, the 1% annual chance flow listed in the current FIS Report is 40,000 cfs. The drainage point that is located at the confluence with Fall River Tributary 3 closely corresponds to the location listed in the FIS Report for Neodesha. However, the study done for the effective FIS Report did not include several large storm events that occurred after the study was conducted; including the 2007 storm, which had a peak discharge of 77,800 cfs at the Fredonia gage. In addition, the period between the construction of the Fall River Lake dam and the time of the previous study was relatively short, lacking a significant period of record for the study. This provides confidence that the higher flows are accurate and appropriate.

VERDIGRIS RIVER

The effective FIS Report for Neodesha, Kansas lists flows for the Verdigris River at the St. Louis and San Francisco Railroad, indicating a drainage area of 1,224 square miles. The results from the gage analysis of the Coyville, Altoona, and Independence gages were interpolated and extrapolated to produce flows at various locations along the Verdigris River. The Controlled Segment Interpolation Procedure was used for determining the flows along the portion of the Verdigris River that is included in this study, as it was the most appropriate method available for the conditions of the stream. Various methods for interpolation of the flows were analyzed; including the Drainage Transfer Method, the Uncontrolled Segment Interpolation Procedure for one gage, the Controlled Segment Interpolation Procedure for one gage, the Uncontrolled Segment Interpolation Procedure for two gages, the Controlled Segment Interpolation Procedure for two gages, and utilization of localized regression equations.

The Controlled Segment Interpolation Procedure for one gage was utilized to interpolate flows upstream of the Coyville gage and downstream of Toronto Lake. The flows were computed using the procedure previously described; utilizing the flows from only the Coyville gage. The station skew method results were chosen for this gage, as previously described.

The Controlled Segment Interpolation Procedure for two gages was utilized to interpolate flows between the Coyville gage and the Altoona gage. The flows were computed using the following parameters, which are described in Table 4 of the USGS Estimates of Flow Duration, Mean Flow, and Peak-Discharge Frequency Values for Kansas Stream Locations (Reference 14); utilizing the flows from both the Coyville gage and the Altoona gage. The station skew method results were chosen for these gages, as previously described.

$$Q_s = \frac{Q_u(DA_d - DA_s) + Q_d(DA_s - DA_u)}{DA_d - DA_u}$$

Where:

Q_s = peak discharge at the ungaged drainage point of interest, in cubic feet per second

Q_u = peak discharge at the upstream gage location, in cubic feet per second

Q_d = peak discharge at the downstream gage location, in cubic feet per second

DA_s = total area that contributes runoff to the ungaged drainage point of interest, in square miles

DA_u = total area that contributes runoff to the upstream gage location, in square miles

DA_d = total area that contributes runoff to the downstream gage location, in square miles.

The Controlled Segment Interpolation Procedure for one gage was utilized to interpolate flows between the Altoona gage and the confluence with the Fall River. The flows were computed using the procedure previously described; utilizing the flows from only the Altoona gage. The station skew method results were chosen for this gage, as previously described.

The Controlled Segment Interpolation Procedure for one gage was utilized to interpolate flows downstream of the confluence with the Fall River. The downstream extent of this study is the Wilson County line. The flows were computed using the procedure previously described; utilizing the flows from only the Independence gage. The weighted skew method results were chosen for this gage, as previously described.

Table 9, shown on the next page, represents the peak discharges computed as part of this statistical analysis, which incorporates analysis from Verdigris River gages at Coyville, Altoona, and Independence.

Table 9: Statistical Analysis Results for the Verdigris River								
Location	Drainage Area (sq. mi.)	Peak Annual-Chance Discharges (CFS)						
		10%	4%	2%	1%	1%-	1%+	0.2%
Just Downstream of Toronto Lake	722.8	7,358	9,109	10,799	12,869	11,091	15,561	19,591
USGS Gage near Coyville	747.0	7,604	9,414	11,161	13,300	11,462	16,082	20,247
At Confluence with Sandy Creek	857.0	13,084	17,649	21,939	27,107	22,065	38,648	43,514
At Confluence with Buffalo Creek	977.9	19,106	26,701	33,785	42,283	33,719	63,449	69,087
At Confluence with Verdigris River Tributary 15	1,051.5	22,773	32,211	40,996	51,521	40,813	78,547	84,654
USGS Gage near Altoona	1,094.0	24,890	35,393	45,160	56,856	44,910	87,266	93,644
At Confluence with Chetopa Creek	1,190.8	27,092	38,525	49,156	61,887	48,884	94,988	101,930
At Confluence with Fall River	2,095.8	36,669	51,456	64,589	79,701	64,908	108,045	124,057
At Confluence with Salt Creek	2,105.8	36,844	51,701	64,898	80,082	65,218	108,560	124,649

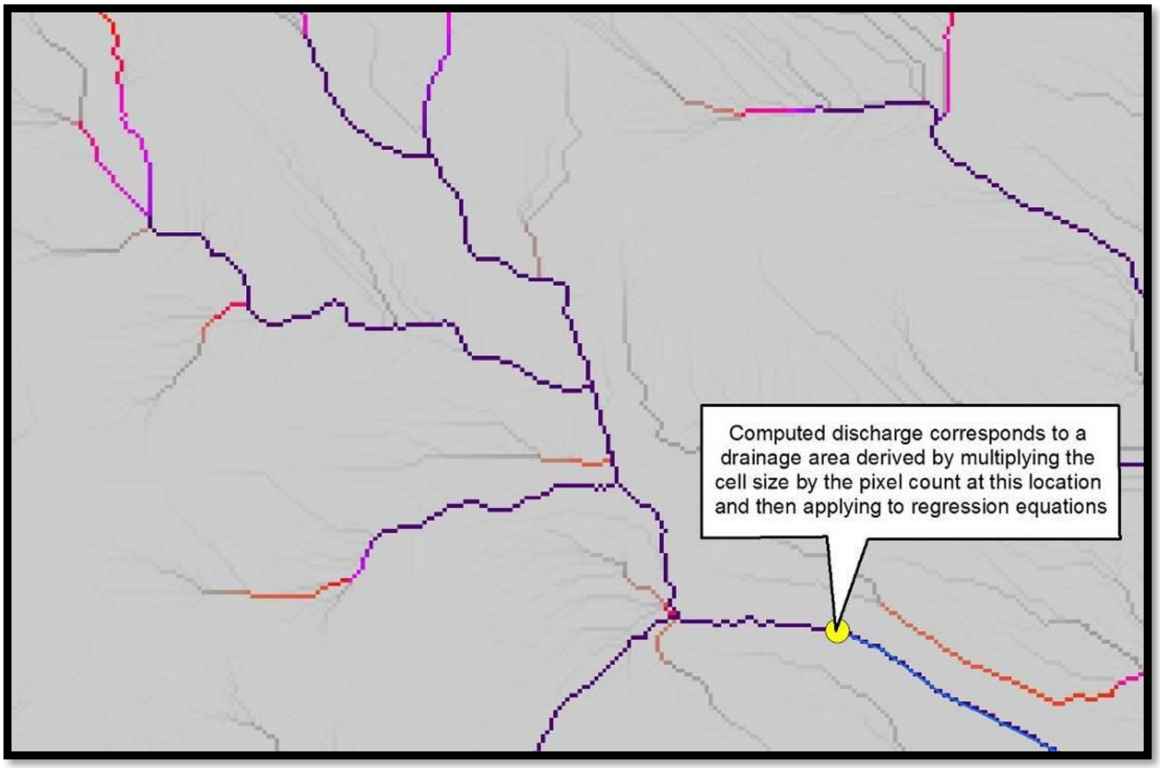
The peak flows for the Verdigris River in the current effective FIS Report for Neodesha, Kansas are lower than the flows described in this statistical analysis. For example, the 1% annual chance flow listed in the current FIS Report is 46,000 cfs. The drainage point that is located at the confluence with Chetopa Creek closely corresponds to the location listed in the FIS Report for Neodesha. However, the study done for the effective FIS Report did not include several large storm events that occurred after the study was conducted; including the 2007 storm, which had a peak discharge of 63,600 cfs at the Altoona gage. In addition, the period between the construction of the Toronto Lake dam and the time of the previous study was relatively short, lacking a significant period of record for the study. This provides confidence that the higher flows are accurate and appropriate.

Approximate Hydrologic Analysis

The hydrology for the Zone A streams that are not modeled by a detailed hydrologic method was developed using USGS Rural Regression Equations for Kansas.

To prepare the drainage network, the scoped streams were adjusted based on LiDAR elevation data and aerial imagery obtained through the Kansas Data Access and Support Center. A flow accumulation grid was developed from the LiDAR data which provides a "pixel count" at desired flow change locations that represents the number of pixels flowing into it. A simple calculation is used to convert this pixel count into square miles. Figure 8 illustrates how the drainage points correspond to the flow accumulation grid.

Figure 8: Regression Analysis Discharge Calculation Example



The drainage points were located using automated processes along the stream centerline, generated from the DEM. The points were intersected with the accompanying flow accumulation grid to establish a contributing drainage area. Initial drainage points were generated every 300 feet along the stream network. Flows for the 1% annual chance storm event were then calculated for each drainage point, based on the USGS Rural Regression Equations for Kansas (Reference 1).

1) For larger drainage areas: $Q_{1\%}=1.16(CDA)^{0.462}(P)^{2.250}$
2) For smaller drainage areas: $Q_{1\%}=19.80(CDA)^{0.634}(P)^{1.288}$

Where:

Contributing Drainage Area (CDA) = is the total area that contributes runoff to the stream site of interest, in square miles.

Precipitation (P) = average mean annual precipitation for the subbasin, in inches.

The intersection of the two regression equations is used to determine the contributing drainage area in which to transition from the smaller drainage area equation to the larger drainage area equation. This varies from the documented threshold of 30 square miles due to the discontinuity that occurs between the two sets of equations at this transition.

After flows were developed using the previously described equations, the drainage point file was filtered to produce the final drainage point file that represents points at or approximately at a 10% change in flows. To establish flow change location; filtering begins at the most upstream drainage point and subsequent downstream drainage points are evaluated. The next flow change location is set to the larger of drainage point values where their percentile difference relative to previous flow value envelops a 10% change. The process is repeated until the end of the stream is reached. The peak flows from the Little Cedar Creek watershed and Salt Creek watershed HEC-HMS models were compared to the flows calculated using the USGS Rural Regression Equations for Kansas. The flows from the HEC-HMS models fall slightly under the Kansas Regression Flows, but are still considered to be within an appropriate tolerance range. Therefore, it was concluding that the Kansas Regression Equations were suitable to use for the Zone A streams not modeled by a detailed hydrologic method. The USGS Rural Regression Equations for Kansas are as follows:

1) For larger drainage areas:
   $Q10 = 0.039\ (CDA)^{0.480}\ (P)^{2.931}$
   $Q25 = 0.195\ (CDA)^{0.469}\ (P)^{2.603}$
   $Q50 = 0.508\ (CDA)^{0.465}\ (P)^{2.411}$
   $Q100 = 1.160\ (CDA)^{0.462}\ (P)^{2.250}$

2) For smaller drainage areas:
   $Q10 = 1.224\ (CDA)^{0.611}\ (P)^{1.844}$
   $Q25 = 4.673\ (CDA)^{0.622}\ (P)^{1.572}$
   $Q50 = 10.26\ (CDA)^{0.628}\ (P)^{1.415}$
   $Q100 = 19.80\ (CDA)^{0.634}\ (P)^{1.288}$

Where:

Contributing Drainage Area (CDA) = is the total area that contributes runoff to the stream site of interest, in square miles.

Precipitation (P) = average mean annual precipitation for the subbasin, in inches.

The intersection of the two regression equations is used to determine the contributing drainage area in which to transition from the smaller drainage area equation to the larger drainage area equation.

Since there is no USGS Kansas Regression Equation for the 0.2% annual chance storm event, Regression Equations for the 0.2% annual chance storm event were determined by an extrapolation procedure that utilizes the other USGS Kansas Regression Equations.

The peak flows for the 1% minus and 1% plus annual chance storm events were determined using the upper and lower limit model standard error of prediction for the 1% annual chance USGS Rural Regression Equations for Kansas (Reference 1). The upper limit model standard error of prediction is +71% for the smaller drainage areas and +47% for the larger drainage areas. The lower limit model standard error of prediction is -44% for the smaller drainage areas and -32% for the larger drainage areas.

Peak flows were then calculated for each drainage point within the filtered points file that was generated for the approximate Zone A streams, using the Kansas Regression Equations for the 10%, 4%, 2%, 1%, 1% Minus, and 1% Plus annual chance storm events and the extrapolated 0.2% annual chance storm event.

CONCLUSION

As a result of this hydrologic analyses, peak discharges have been developed for the 10%, 4%, 2%, 1%, 1%-minus, 1%-plus and 0.2% annual chance storm events for the detailed Zone AE streams, the enhanced Zone AE streams, and the approximate Zone A streams. Peak discharges for the detailed Zone AE streams and the enhanced Zone AE streams, developed by the enhanced hydrologic analyses described in this report, are represented in Table 10 – Summary of Discharges.

TABLE 10 – SUMMARY OF DISCHARGES

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (mi^2)	PEAK ANNUAL-CHANCE DISCHARGES (CFS)						
		10%	4%	2%	1%	1%-	1%+	0.2%
Fall River								
At US Highway 400	885	24,109	34,324	43,793	55,099	44,042	82,050	90,475
Little Cedar Creek								
At Mouth	13.1	4,439	6,148	7,250	8,708	7,326	10,295	12,747
At KS Highway 47	12.6	4,447	6,130	7,201	8,692	7,278	10,271	12,738
Salt Creek								
At Confluence with Salt Creek Tributary 1	19.0	5,437	6,952	8,197	9,423	8,296	10,757	12,745
At Washington Street	16.6	5,126	6,572	7,761	8,921	7,858	10,116	11,932
At 1150 Road	13.7	3,647	4,107	4,429	4,699	4,454	4,948	5,288
Salt Creek Tributary 1								
At Mouth	2.03	754	1,055	1,330	1,585	1,352	1,829	2,177
At Cement Plant Road	1.30	518	761	999	1,233	1,019	1,509	1,917
At US Highway 400	0.45	328	439	535	631	543	742	908
Salt Creek Tributary 3								
At Mouth	1.51	1,403	2,391	3,269	4,196	3,342	5,236	6,714
At South Kansas and Oklahoma Railroad	1.33	985	1,254	1,495	1,718	1,515	1,960	2,409
At 15th Street	0.56	505	676	823	970	835	1,140	1,395
Verdigris River								
At Union Pacific Railroad	1,191	27,092	38,525	49,156	61,887	48,884	94,988	101,930

Disclaimer: As mapping tasks are completed, the potential for minor changes to the information submitted in the hydrology submission and within this report may become necessary. The data provided in this submission and report may not be completely representative of the hydraulics used to produce the final map product. Therefore, this report and the hydraulics submission should be considered as draft. This submission should be considered a complete step in progress but not necessarily the final product since the post preliminary process is not yet completed and the floodplain maps are not yet effective.

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